EFFECT OF ROCK TEXTURES ON SLOPE STABILITY: EXPERIMENTAL STUDY

By

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ABSTRACT

The research reported in this thesis focused on the effects of rock texture on the slope stability (open cast mines, quarries and slides of roads and rivers). Artificial rocks were made from cement, sands and basalt particles of different sizes in order to produce artificial rock differ in their textures.

Three kinds of rock textures were designed to investigate the effects of the rock texture on the slope stability of benches at various angles of slope and various bench heights. Crushed basalt samples were collected from a basalt quarry in Sabaloga, these crushed basalt samples were classified to produce three sizes coarse, medium, and fine which were used to produce coarse, medium and fine artificial rock textures.

It was found that the benches would be more stable at grains having sizes less than 5 mm for slope angle less than 75 degree. But when the grain sizes coarser than 5 mm, the benches would not be stable at this angle as a result of both tension and shear stresses depending on the bench height.

The relationships between the failure loads and grain sizes yielded second order equations which are shown graphically as curves.

المستخلص

يركز هذا البحث على تأثير نسيج الصخور على إستقرار المنحدرات (مصاطب المناجم المكشوفة ، المحاجر وجوانب الطرق والأنهار). تم تصنيع صخور إصطناعية من الرمل وحصى بازلت بأحجام مختلفة وأسمنت بورتولاندي عادي للحصول علي صخور صناعية ذات أنسجة مختلفة.

ثلاثة أنواع من الصخور مختلفة الأنسجه صممت لدراسة تأثير أنسجة الصخور علي إستقرار ميل المصاطب عند زوايا الميل المختلفة وإرتفاع مصاطب مختلفة، تم تجميع عينات البازلت المكسرة من محجر البازلت بالسبلوقة ثم صنفت هذه العينات للحصول على ثلاثة أحجام: خشنة ، متوسط الخشونة وناعمة وذلك لإنتاج ثلاثة نماذج لأنسجة الصخور الإصطناعية الخشنة، المتوسطة الخشونة والناعمة.

ولقد أوضحت النتائج الآتي : أن المصاطب تكون أكثر إستقراراً عندما يكون حجم الحبيبات المكونة للصخور أقل من 5 ملم وزاويا ميول المصاطب أقل من 75 درجة ، أما عندما يكون حجم الحبيبات أكبر من 5 ملم تكون اقل إتزاناً عند هذه الزاوية نتيجة لتعرضها لإجهادات الشد والقص معتمدةً علي إرتفاع المصطبة.

كما وجد أن العلاقة بين قوة تحمل الصخر تتناسب تناسباً عكسياً مع حجم البنيات المكونة له كما يظهر ذلك بيانياً.

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CHAPTER ONE
INTRODUCTION

1.1 Problem background:-

Movements of rock masses known as landslides are caused by the direct or indirect action of gravity on unstable materials. They are important factors in the extraction of rocks or ore minerals from quarries and mines and frequently occur at stope faces or tunnel walls and roads walls leading to the failure of quarries or mine benches or tunnel and roads walls.

Landslides may be divided into two broad groups according to the nature of the movement involved:

(a) slides, in which a surface of slides is present, separating the moving mass from the stable ground.

(b) flows , in which there is no surface sliding but movement takes place by continuous deformation ; the motion is generally less rapid than in the case of slides , and may be very slow indeed but there are transition from one group to the other.

In open casts or quarries excavation, the nature of rocks and mineral ores through which excavation is made will govern questions of construction such as the slope of benches, tunnels or the slope of the hills, method and rate of excavation and to a great extent its cost. Thus, vertical sides are possible in massive bedded hard strata, horizontal or gently dipping, with few joints, or in strong igneous rocks. In other sedimentary rocks or mineral ores it is necessary to take into account the relation between their structures, textures and the direction of cuttings.

In all cases of rocks or of mineral ores cuttings are rarely made with vertical walls and the sides or faces are generally sloped symmetrically at a safe angle.

Open pit mines or quarries are large rock excavations that are usually intended mainly to strip away overburden materials from ores or rocks. Their designs, that is, the choice of angles for stops, widths of benches, and overall shape are now integrated with other mining cost factors to achieve maximum profit, too flat a slope will mean extra excavation and extra waste rock; but to steep a slope will increase the number of lost time haulage road blockages and accidents.

Most of the slopes of an open pit or a quarry are temporary since the pit or quarry is ever enlarging .Simple instrumentation and quick response to signs of slope instability have allowed mining industries to work safely with slopes that would be judged too steep for civil engineering excavations of comparable size.

In soft rocks and ores, like shales, hydrothermally altered zones, and deeply weathered granites, design of safe slopes is an extension of soil mechanics theory, since such materials tend to fail by slumping or sliding through the body of rock itself.

In most hard rocks and in some of softer rocks as well, preexisting discontinuities control the avenues of rock movement so modes of slope failure occur that are not usual in soils. Special methods for analyzing these structurally controlled failure modes have been devised by workers in rocks mechanics.

The effects of kind of rocks, the water content, the structural geology, the bench design parameters (such as height of benches and their widths), specific gravity of rocks, etc .were studied by many workers in rock mechanics as well.

It is believed that the texture and the characteristics of grains of minerals of rocks would play significant roles on the slope stability during extraction of rocks from their deposits. Hence, physical models were made of cement, sands having small amounts of iron oxides, water and basalt particles. The basalt particles used of three sizes that is small, medium and large particles, in order to study the effect of rock grain sizes on the slope stability. Also, the effects of the height of the benches and their slopes on the stability of these benches were studied, and the results are present in this article. The modes of failure for each case are also shown.

1.2 Objectives of study:

The main objective of this study is to determine the slope stability of rock in mines depending physical properties of rock, also to investigate the relations between rock textures and the following:

1/ Angle of slope

2/ Bench height

3/ Spacing

4/ Burden

CHAPTER TWO
LITERATURE REVIEW

2.1 Sliding Failure:-

In open pit mining, the optimum slope angle is usually one that maximizes overall slope angle and minimize the amount of waste stripping. At the same time, it must manage the risk of overall slope instability, and provide safe and efficient movement of personnel, equipment and materials during mining operation (Wyllie and Mah, 2004).

The objectives of economy and safety, as a rule, involve the maximization of the angle of inclination of the slope while assuring stability. Stability assurance requires an appreciation for the potential modes of failure and rock sliding.

2.1.1 Sliding Failure Mechanisms:-

Figure 2.1 illustrates seven failure mechanisms that may be associated with the sliding failure mode. While other mechanisms are conceptually possible, the seven mechanisms illustrated are representative of those mechanisms most likely to occur in the nature or in mining industries and civil work. The following discussions provide a brief description of the conditions necessary to initiate each of these engineering sliding mechanisms which was given by Brown (Brown, 1994).

***a. Single block/single sliding plane*.** A single block with potential for sliding along a single plane which represents the simplest sliding mechanism, (see Figure 2.1a).

b. Single block/stepped sliding planes. Single block sliding along stepped planes is possible in cases where a series of closely spaced parallel joints strike approximately parallel to the excavation slope strike and dip toward the excavation slope. The parallel joints may be or may not be continuous, (see Figure 2.1b).

***c. Multiple blocks/multiple sliding planes*.** Multiple blocks, sliding along multiple planes are the most complicated planar type of sliding, (see Figure 2.1c).

***d. Single wedge/two intersecting planes*.** Single wedge sliding can occur in rock masses with two or more sets of discontinuities whose lines of intersection are approximately perpendicular to the strike of the slope and dip toward the plane of the slope, (see Figure 2.1d).

e. Single wedge/multiple intersecting planes. The conditions for sliding of a single wedge formed by the intersections of at least two discontinuity sets with closely spaced joints are essentially the same as discussed in **d** above, (see Figure 2.1e).

***f. Multiple wedges/multiple intersecting planes*.** Multiple wedges can be formed by the intersection of four or more sets of discontinuities, (see Figure 2.1f).

***g. Single block/circular slip path*.** Single block sliding failures along circular slip paths are commonly associated with soil slopes. However, circular slip failures may occur in highly weathered and decomposed rock masses, highly fractured rock masses, or in weak rock such as clay, shales and poorly cemented sandstones, (see Figure 2.1g).

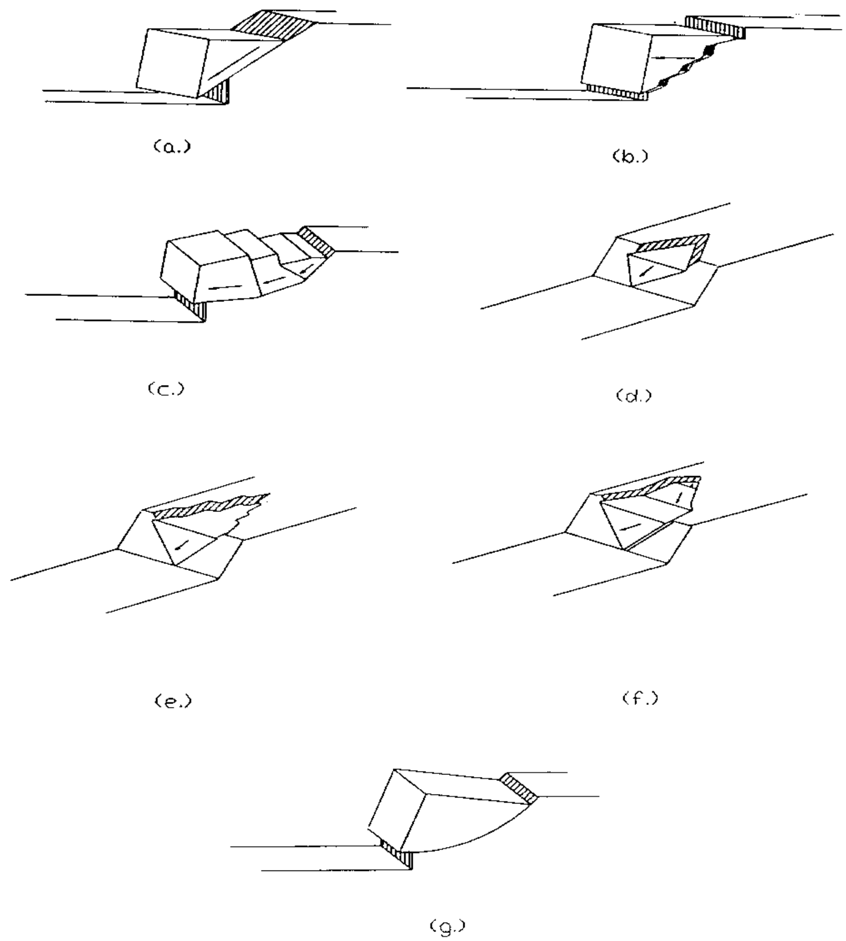


Figure 2.1. Failure sliding mechanisms (After Brown, 1994)

2.1.2 Sliding Failure Modes:-

(I) Toppling Failure:

Toppling failure involves overturning or rotation of rock layers. Closely spaced, steeply dipping discontinuity sets that dip away from the slope surface are necessary prerequisites for toppling. In the absence of cross jointing, each layer tends to bend down slope under its own weight thus generating flexural cracks. If frequent cross joints are present, the layers can topple as rigid columns. In either case, toppling is usually initiated by layer separation with movement in the direction of the excavation.

Layer separation may be rapid or gradual. Rapid separation is associated with block weight and/or stress relief forces. Gradual separation is usually associated with environmental processes such as freeze/thaw cycles, (Brown, 1994).

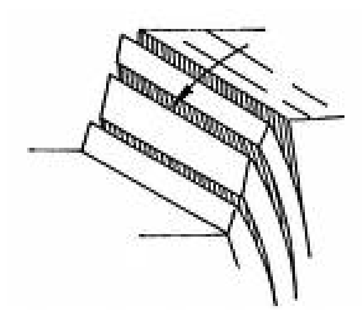


Figure 2.2 Toppling Failure Mode

(II) Sloughing Failure:

Sloughing failures are generally characterized by occasional rock falls or localized slumping of rocks degraded by weathering. Rock falls occur when rock blocks become loosened and isolated by weathering and erosion.

Some rocks disintegrate into soil-like material when exposed to repeated wetting and drying cycles. This material can fail in a fashion similar to shallow slump type failures commonly associated with soil slopes. Both rock falls and localized slumping constitute more of a maintenance problem than a major slope instability threat. However, slopes in sedimentary rock that are interbedded shale layers can experience major slope failures initiated by localized deterioration of the shale layers. Deterioration of the shale layers leads to the undermining and hence failure of the more competent overlying layers, (Brown, 1994).

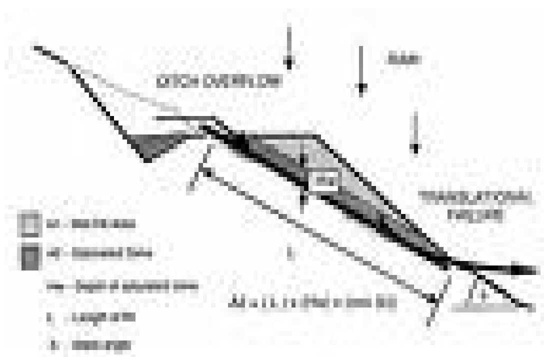


Figure 2.3 Sloughing Failure Mode

(III) Raveling:

Weathering of material and expansion and contraction associated with freeze-thaw cycles are principle causes of raveling. This type of failure generally produces small rock falls, not massive failures, (Girard, 2001).

2.2 Factors affecting the Slop Stability:-

2.2.1 Geologic and geotechnical factors:-

The stability of rock slopes is often significantly influenced by the structural geology of the rock in which the slope is excavated .structural geology refers to naturally occurring breaks in the rock such as bedding planes, joints and faults ,which are generally termed discontinuities .The properties of discontinuities relative to stability include orientation, persistence, roughness and infilling. The significance of discontinuities is that they are planes of weakness in the much stronger, intact rock so failure tends to occur preferentially along these surfaces.

Alternatively, discontinuities may only indirectly influence stability where their length is much shorter than the slope dimensions, such as an open pit mine slope where no single discontinuity controls stability. However, the properties of the discontinuities will affect the strength of the rock mass in which the slope is mined.

Almost all rock slope stability studies should address the structural geology of the site, and such studies involve steps as follows .first, determine the properties of the discontinuities, which involves mapping outcrops and existing cuts, if any, and examining diamond drill core, as appropriate for the site conditions. Second determine the influence of the discontinuities on stability, which involves studying the relationship between the orientation of discontinuities and the face, (Wylli and Mah, 2004).

The overall purpose of a geological mapping program is to define a set or sets of discontinuities or a single feature such as a fault which will control stability on a particular slope. For example, the bedding may dip out of the face and from a plane failure or a pair of joint sets may intersect to from series wedges. It is common that the discontinuities will occur in three orthogonal sets (mutually at right angles), with possibly one additional set. It is suggested that four sets is the maximum that can be incorporated into a slope design, and that any additional sets that may appear to be present are more likely to represent scatter in the orientation of the sets. Discontinuities that occur infrequently in the rock mass are not likely to have a significant influence on stability of the overall slope and so can be discounted in design. However, it is important to identify single feature such as a through- going, adversely orientated fault that may be a controlling feature for stability, (Wylli and Mah, 2004).

A study by Patton and Deere was made on the geologic factors which controlling the stability of rock slope in open pit. They have shown little doubt that the through-going faults and shear zones and the intersection of such structures are the most significant parameters, because of their continuity they can influence large areas of pit slope and often more than pit sides. In addition, geologic displacements along faults and shear zones have led to the crushing or overriding of most irregularities in at least one direction so that low residual shear strengths are often applicable rather than the higher strengths associated with more irregular rock surfaces, (Patton and Deere, 1970).

Chemical alteration of the surrounding rock and the frequent presence of breccia and clay gouge are also commonly associated with faults and shear zones .These could lead to a decrease in the influence of surface irregularities as the intact material is more readily sheared off. Finally, the presence of clay gouge adjacent to the polished or smooth faults surfaces could explain the unusually low strengths encountered in the laboratory for soil – rock surfaces which are developed at small displacement and are applicable to the field problem .In spite of their size and continuity, the major faults and their inter sections are not always readily seen until after the slope failure develops, (Hoek, 1970).

The stability of slopes in materials which would normally be cased as rock is controlled by structural discontinuities such as faults, joints and bedding planes in the rock mass. Not only do these discontinuities provide surfaces open which sliding can take planes but they also control the flow of ground water which has a very important influence on slope stability. The mapping of these structural features represents a major problem in slope stability engineering and development of mapping techniques is lagging far behind developments in other areas of slope stability analysis, (Hoek, 1970).

Advances in our ability to design and control slopes will depend, to a large extent, upon our ability to devise more effective structural mapping techniques .it is also important that care is exercised in the interpretation of shear strength test and it is suggested that, when ever possible, such test results be checked against actual strengths characteristics as determined from the back analysis of slope failures. (Hoek, 1970).

2.2.2 Ground water:-

Ground water occupying the fractures within a rock mass can significantly reduce the stability of a rock slope. Water pressure acting within a discontinuity reduces the effective normal stress acting on the plane, thus reducing the shear strength along that plane. Water pressures within discontinuities that run roughly parallel to a slope face also increase the driving forces acting on the rock mass, (Morgenstern, 1970).

Morgenstern has concluded his study that water pressure reduces the normal of active stress in rock and the resistance to shear drops accordingly. Drainage a rock mass will reduce the water pressures and increase the shear strength. Hence aeration, drainage stabilization scheme requires knowledge of the water pressure distribution in the rock mass.

The factors governing the flow of water through discontinuities in a rock mass have become relatively well understood in the past few years. The theory for predicting the pressure distribution with confidence.

This data is difficult to obtain. Hence it is recommended that reliance be placed on water pressure observed directly in the rock mass. This information is directly useful in stability analysis is and provides a base for predicting the influence of for the excavation or drainage on slope stability, (Morgenstern, 1970).

The analysis of groundwater quality showed that change in water quantity caused the variation of water quality. Rotaru and Raileanu, 2004, have analyzed groundwater system respecting to hydrogeological aspects and put an emphasis on the hydrogeological activities and methodology of a reliable, quality determination of the groundwater regime in the landslide body as a basis for geotechnical modeling of slope stability, in order, to prepare optimum measures for protection. According to the geological properties, aquifers was be simplified as a one-layer aquifer model. A two dimensional model is adopted to simulate the fluctuation of groundwater level.

The groundwater in alluvial deposit plain is seriously affected by over-pumping. Groundwater level varied seasonally, by pumping amount and precipitation, (Rotaru and Raileanu, 2004).

However, one-layer two-dimensional plane model matched well with actual observations of alluvial deposit plain. It could be extended to predict the fluctuation of groundwater level, and gives a reference to guide pumping. Because of the over-pumping in the alluvial deposit plain, the groundwater in shallow aquifer and medium aquifer would be polluted by water intrusion to a different degree. The pollution in the area near the lake would be more serious than that in the inland area. It is remarkable as the draw down due to over - pumping in shallow aquifer and medium aquifer induced to old water seepage from deep aquifer, (Rotaru and Raileanu, 2004).

The groundwater in such area may be affected more strongly by fossil water than lake water or river water. Therefore in order to utilize and protect groundwater resources, more detail observations and study about the groundwater system would be necessary in the future.

On the basis of a stability analysis and with a view to all preliminary geological, hydrogeological and hydrodynamic observations, the following conclusions and recommendations have been made by Rotaru and Raileanu, 2004.

With regard to the latent risk that the existing natural state of balance may be disturbed on the whole analyzed slope, any inappropriate big cutting of the ground, say, for example for making terraces for vine rows, for local roads, for local building construction, then landfills on some sections or humus topsoil redistribution may contribute to disturbing the balance and may intensify the sliding process depending on the current groundwater table. More extensive changes in the slope morphology will imminently also bring about changes in the natural groundwater table in the treated sections and on the whole grounds, (Rotaru and Raileanu, 2004).

Since lowering of the first aquifer groundwater table from the existing average values and its maintenance on that average value will lead to satisfactory stability of the analyzed slope (one of rational measures of protection) it will be indispensable to define by appropriate quantitative hydrodynamic analysis, the most optimum technical solution for ground drainage.

. In order to arrive at a reliable definition of the groundwater regime it will be necessary to continue observations for at least one hydrological year and compare results with the vertical balance parameters. Simultaneously with groundwater table observations it will be necessary to measure both active and incidental physical movements of ground on the survey network and conduct an adequate periodical analysis of stability particularly checking the effects of extreme groundwater rises in spring. The ground stability in the narrow zone of the building area requires a specific detailed analysis of the existing drainage system around and under the houses. The groundwater table observations in the water bodies close to the building area may show that the drainage system is shallow and most probably sealed, (Rotaru and Raileanu, 2004).

Accurate slope-stability analysis of pre-existing landslides and adjacent, potentially landslide-prone slopes requires a realistic estimation of maximum ground-water levels. Previous researchers have documented a long-term rise in ground-water levels since the 1960s in unconsolidated deposits near recently active landslides. Ground-water levels rose mostly during and immediately prior to a wet cycle that started in 1967. Between 1967 and 1998, cumulative excess precipitation totaled about 26 inches in Salt Lake City, coincident with a long-term rise in ground-water levels by as much as 25 feet period fluctuated to historically high levels and active landsliding by the late 1990s, (Ashland, et. al., 2006).

In the Sherwood Hills landslide in Provo, rising ground-water levels in the lower part of the landslide, accompanied by first declining and then rising levels in the upper part of the slide, preceded the onset of damaging movement in 2005, suggesting a dynamic ground-water-level-fluctuation model for landslide reactivation, (Ashland, et. al., 2006).

Conventional methods of slope stability computation are found to yield different results. This difference may be due to improper estimation technology of ground water pressure forces. A ground water factor is of special importance in clay slope stability. Unless clay permeability and mechanical properties are taken into account in computations, significant errors may arise, (Zdankus and Stelmokaitis, 2007).

Dependence of clay permeability on both hydraulic gradient and zero permeability, when the gradient is lower than the threshold, is typical of clay. For this reason a drawdown curve in clay slope is short and steep, it frequently coincides with the slope surface profile. Sudden loading of the slope and increment in clay compression lead to twice as much decrement in cohesion and significant reduction in internal ground friction. Neglect of these peculiarities in clay slope stability computations results in too optimistic, regrettably, wrong results, (Zdankus and Stelmokaitis, 2007).

To avoid these errors a drawdown curve in clay slope stability computations should coincide with the slope profile. Clay cohesion and an internal friction angle have to be determined using the data obtained in testing clay of undisturbed structure under undrained, unconsolidated conditions. In computation, a hypothetic clay slope sliding body should be divided into vertical slices and water pressure forces should be computed for each slice separately, not for all the body, as it is accustomed to do, (Zdankus and Stelmokaitis, 2007).

2.2.3 Blasting and earthquakes:-

It has been suggested by Richard, et. al, 1970, that one of the major problems that arises from the excavation of rock slope is that the very material trying to keep stable is seriously damaged to the very high seismic shocks caused by blasting. Most open pit operators are familiar back break form blast, but most people only consider the visible breakage behind the row of holes of the blast. But how far back dose the damage really extend, when damage is defined as actual deterioration of the rock structure it self? In one recent example, a tunnel was driven in the supposedly highly competent rock of an open pit wall, spilling had to used to drive the tunnel in 80 m , closest to the pit wall .It was felt that at least 50% of this damaged rock was caused by the seismic shock form the large blasts in the pit. The slopes in an open pit need the most protection from the effects of blasting at the final walls since it is here that steepening is most likely to be carried out and hence, the best rock characteristics are required, (Richard, et. al, 1970).

Chain and Talhi, 2005, have analyzed the behavior of the rock after blasting. They have reasoned that into tow groups : First, rock of low behavior where the ground is supposed to be in an elastic plastic model, in which, is formed in the up grade bench three zones: zone of large discontinuities (the plastic zone near the drilling),the zone of small discontinuities (second plastic zone) and beyond it an elastic zone develops .The bench slope stability requires the reduction of the first plastic zone .The radius of that zone is determined as a function to the static strain energy as well as dynamic energy resulting, respectively, from the static pressure in the drilling hole and from the introduced explosive quantity . Secondly, the radius is calculated as function to the explosive quantity charge into the hole and equally on site.

Casale, et. al, 2008, have discussed operations aimed at creating a safer natural or man made rock slope by artificially inducing the displacement of unstable elements by blasting.

The demolition of unstable rock volumes by explosives to reduce the risk of collapse from rock slopes is a technique which can be cheaper and faster when compared to the use of reinforcing elements, net fences and/or protection embankments.

The examples that have been discussed by them show that with an accurate preliminary geotechnical study and with a detailed blast design, the drill and blast option can be an appropriate solution to complex and difficult stability problems on rock slopes.

The variability of local situations (geometry, topography, joint set layout, etc.) prevents the formation of a general schematic blasting round but the design must be therefore developed by blasting specialists taking into consideration the following criteria, Casale, et. al, concluded that:

(i) Limitation of the instantaneous charge to be blasted for each delay to avoid blast vibrations that could unstabilize an existing rock portion on a nearby slope.

(ii) Use of a reliable triggering system since a failure in the detachment of the rock mass or the presence of unblasted charges inside the muck can create very critical and dangerous situations that can be difficult to be corrected;

(iii) Fragmentation of the blasted material into small pieces that can easily be mucked without causing excessive damage to the existing infrastructure, which is often protected using a soft soil mattress;

(iv) Prevention and control of fly rock:

The key challenge is usually the definition of the geometrical and geostructural condition of the volumes to be removed in such a way as to correctly locate the round holes. Normally a high specific drilling value is used both to reduce the quantity of instantaneous charge and to obtain a high fragmentation. The powder factor criteria are usually used for the design of the charge. The used powder factor values are similar to those currently employed in quarries, (Casale, et. al, 2008).

Microzonation studies for seismic hazard have many uses (Ozcep and Kaya, 2007). They can provide input for seismic design, land use management and estimation of the potential for liquefaction and landslides. Earthquake-induced landslides have caused tremendous amounts of damage throughout history. In many earthquakes, landslides have been responsible for as much or more damage than all other seismic hazards combined. When an earthquake occurs, the effect of earthquake-induced ground shaking is often sufficient to cause failure of slopes. Resulting damage can range from insignificant to catastrophic depending on geometric and material characteristics of the slope. The amplified motions have devastating effects on structures with periods close the site periods. The site condition includes rock properties beneath the site to depths of up to about few kilometers, the local site conditions, and the topography of the site. In this study, soil amplifications and slope stability analysis will be evaluated in microzonation studies, (Ozcep and Kaya, 2007).

2.3 Slope monitoring system:

The management of risks associated with slope instability is an essential process in the safe and economic operation of open cut mines. The ‘slope stability radar’ (SSR) has been developed to better manage those risks, (Harries, et. al, 2005). The SSR remotely scans rock slopes to continuously measure any surface movement and can be used to detect and alert users of wall movements with sub millimetre precision. The high level of movement precision and broad area coverage of the rock face can allow for a better understanding of the geomechanics of slope deformation, including magnitude of potential failures and additional warning time of impending instability. Additionally, radar waves adequately penetrate through rain, dust and smoke to give reliable measurements, 24 hours a day, (Harries, et. al, 2005).

The SSR is alsoa state-of-the-art development for monitoring slope movement in open pit mines. It offers unprecedented sub-millimetre precision and broad area coverage of wall movements through rain, dust and smoke. The real-time display of the movement of mine walls has allowed continuous management of the risk of slope instability at a mine operations level. There are two key roles where mines are now using the slope stability radar: (Harries, et. al, 2005)

1. Safety Critical Monitoring: The radar is used during mining production as a primary monitoring tool of a designated unstable slope.
2. Campaign Monitoring: The radar is moved around the mine in a repeatable manner to compare movements at each site over an extended time, and determine problematic areas. Campaign monitoring in this manner is often used in metalliferous mines until determination of developing failure is observed.

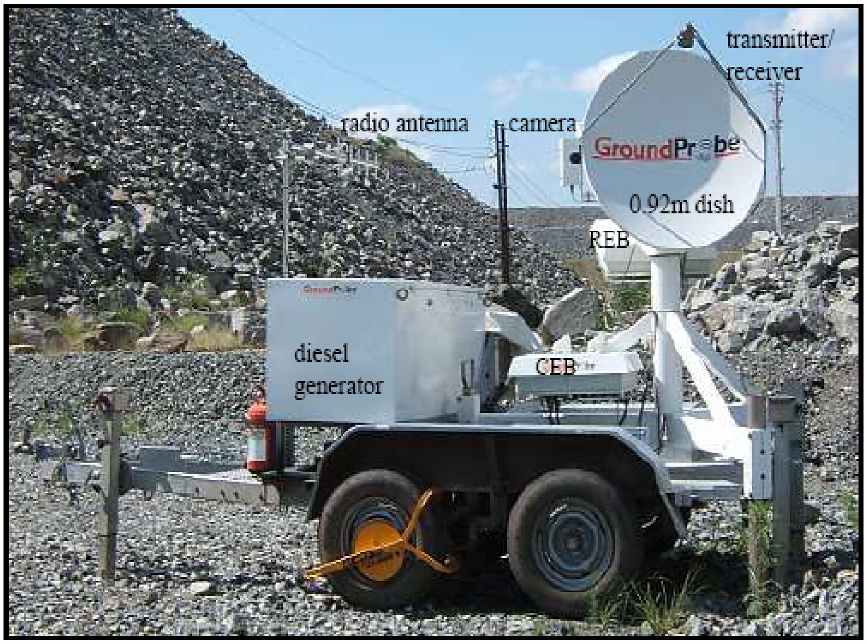


Plate 2.1 Slope Stability Radar

Little, 2005, has reported that Potgietersrust Platinum's Ltd (PPRust) is Anglo Platinum's only open pit operation. The major slope stability concern at PPRust is rapid, small-scale brittle failure on the west wall of Sands loot open pit. In order to improve safety and mine more economically, a comprehensive slope monitoring strategy has been implemented. In the last 3 years four new state-of-the-art monitoring systems have been installed namely an ISSI micro seismic monitoring system, a GeoMoS automated prism monitoring system, prism less Riegl laser scanners and a GroundProbe slope stability radar (SSR).

Groundwater monitoring and visual monitoring have also been improved over the same time period. To complement the visual inspections, Sir Vision digital photogrammetry is now used for predicting where future failures may occur. All these monitoring tools provide primary monitoring which is used to identify high risk areas. The SSR is then set up in that area to provide early warning of failure so evacuation can be successfully done. Fault tree analysis has proved that with this comprehensive slope monitoring strategy the geotechnical risk at PPRust is greatly reduced, allowing mining to continue safely and economically in challenging conditions.

Girard and McHugh, 2000, have summarized others methods to measure and monitor slopes. They are:

(i) **Tension Crack Mapping**: The formation of cracks at the top of a slope is an obvious sign of instability. Measuring and monitoring the changes in crack width and direction of crack propagation is therefore required to establish the extent of the unstable area. Existing cracks should be painted or flagged so that new cracks can be easily identified on subsequent inspections. Measurements of tension cracks may be as simple as driving two stakes on either side of the crack and using a survey tape or rod to measure the width of the crack.

Another common method for monitoring movement across tension cracks is with a portable wire-line extensometer (Figure 2.4).

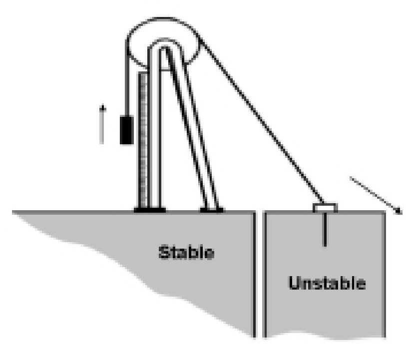


Figure 2.4 Portable wire-line extensometer for monitoring a tension crack.

(ii) Inclinometers: An inclinometer (Figure 2.5) consists of a casing that is placed in the ground through the area of expected movements. The end of the casing is assumed to be fixed so that the lateral profile of displacement can be calculated. The casing has grooves cut on the sides that serve as tracks for the sensing unit.

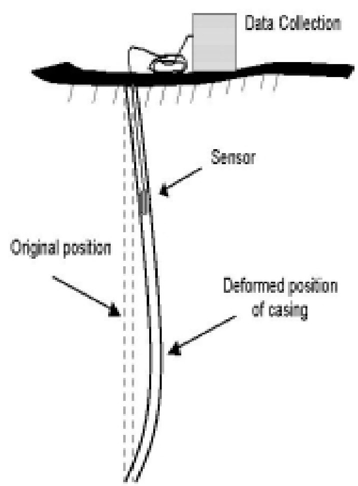


Figure 2.5 Cross-sectional schematic of typical traverse-probe inclinometer system.

(iii) **Borehole Extensometers:** An extensometer consists of tensioned rods anchored at different points in a borehole (Figure 2.6). Changes in the distance between the anchor and the rod head provide the displacement information for the rock mass.

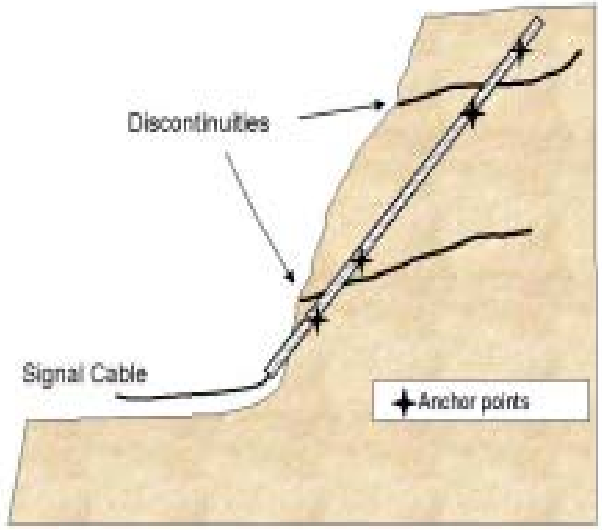


Figure 2.6 Multi-point borehole extensometer**.**

(iv)Survey Network: A survey network consists of target prisms placed on and around areas of anticipated instability on the slopes, and one or more non-moving control points for survey stations. The angles and distances from the survey station to the prisms are measured on a regular basis to establish a history of movement on the slope. It is extremely important to place the permanent control points for the survey stations on stable ground. The surveys can be done manually by a survey crew or can be automated.

2.4 Stabilizing of rock slope:-

The stabilization of slopes is not a design problem since a slope is designed before it is made. For a slope to be stabilized it must already exist.

Golder, 1970, has classed the stabilization for these kinds into:

1- Mechanical strengthening: It can be rock bolting, stressed steel anchors (either bolts or cables) and grouting:

(i)Grouting: assure or an open pit joint can be grouted, but there is no increase in strength until the grout has set. Even after the grout has set there will be little increase in strength if the joint faces are lined with soft material which is not displaced by the grout. This is often the case.

Before the grout has set, the grouting material is a heavy fluid and exerts a pressure which decreases stability. This means that a fissure close to an open cut must not be grouted unless the face is previously bolted into the mass of the rock. For this reason grouting is of limited value in rock slope stabilizing.

(ii) Rock bolting: this is a very generally used method in tunnels in rock to hold up the roof in the zone which has been loosened by blasting and excavation. Steel bolts 10-20 ft. long are inserted into percussion drilled radial holes in the roof and held by an expanding foot at the end of the hole, or alternatively by grouting the steel into the hole. A small amount of pre-stress is put into the bolt by the nut at the outer end.

(iii) Pre stressed cables; in principle these are the same as rock bolts but it is possible to apply considerable pre stress in the cable. This increases the normal pressure across a joint or bedding plane and so the frictional resistance.

2- Change shape of profile: It is obvious that a flat slope is more stable than deeper slope.

One point which must be watched here is that the stability of the steeper slope by the shaping the slope to be adequate.

Under change of shape can also be included removed of rock above a potential surface of failure. This also can increase stability if the rock removed from the right place.

3- Drainage: Generally drainage is a most effective method of increasing stability. However if there is no water in the fissure of the rock then drainage will be ineffective.

That there is no water in the pit does not necessarily mean that there is no water in the rock. As state above, a fault zone filled with impervious gouge can create a zone of high water pressure but no seepage. Further, where the seepage is small, in a hot climate evaporation can remove all water at the rock surface, but nonetheless a water pressure may exist in the rock mass.

In most areas there is water in the ground at some distance below the surface. When a pit is made the horizontal stresses in the ground are relieved and vertical and inclined joints open and water seeps into these joints. This has two related effects:

1) A horizontal water pressure is set in an open vertical joint. This is a disturbing force.

2) A water pressure is set up around any potential surface of failure. The pressure perpendicular to the surface of failure and reduces the effective pressure on this surface, and so the shear strength and the resisting forces and the factor of safety.

CHAPTER THREE
MODEL MATERIALS

3.1 Introduction:

Selection of model material is one of the most time consuming aspects of the engineering model studies. There are an infinite number of model materials that can be used. Each material has its advantages and disadvantages. For the present work, it was suggested that the material should be selected to fulfill the following characteristic:

(i) Behavior of the produced physical models should be similar to the scaled rock type.

(ii) Inexpensive.

(iii) Readily available.

(iv) Workable with respect to material placement and testing.

(v) Reproducible from one batch to the next.

(vi) Easily fabricated.

(vii) The fabricated models should be suitable for the testing machine.

3.2 Selection of the models materials:

Taking the above material characteristic requirements for the choice of model materials, the suitable model materials are as follows:

_ Cement, silica sands, crushed basalt particles and water which were employed to make models are similar to the open cast or quarries benches, (see Plate 3.1).

_ The used cement was Portland cement (OPC).

_ Silica sand having size range from 0.295mm to 0.104mm was washed to remove all clay materials and dirt from it.

_ Drinking tap water was used for molding the models.

_ Basalt particles were screened to give three groups of particles having average sizes 14.3, 7.15 and 3.35mm; these gave three different artificial rocks having coarse, medium and fine texture (see Plate 3.2). While, Table 3.1 shows the physical properties of the used basalt work.

_ The ratios of the materials, i. e., Cement, Sand, Water and Basalt particles, 1: 2.3: 0.6: 5.1, respectively.

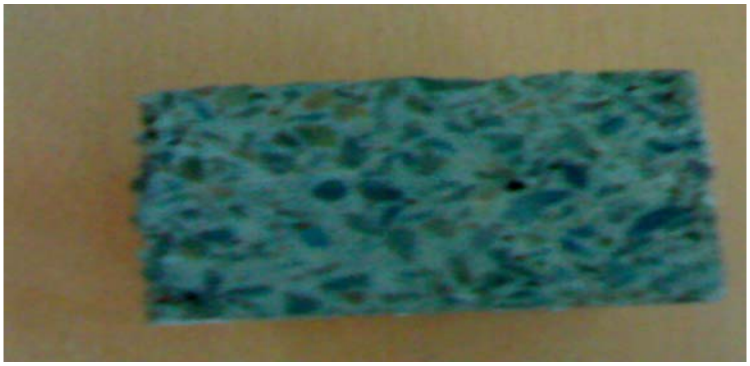
Table 3.1 Basalt rock properties.

Compressive Strength	96 M pa
Tensile Strength	12 M Pa
Point Load Index	86
Specific Weight	2.69 g/cm3

Plate 3.1 shows a few of molded specimens benches.

Course texture rock

Medium texture rock



Fine texture rock

Plate 3.2 the basalt particles sizes which gave three artificial rock having different textures.

In order to complete the analysis of used materials forming the physical models, cubes made of all materials used were tested to determine their strength. Their results are given in table 3.2. However, cubes of the matrix (Mortar) materials of the artificial rock, i. e, cement, sand and water were also made and tested to find out their compressive strength, the results are shown in Table 3.3.

These both types of cubes, concert and mortar were made and tested for the sake of comparison between the obtained results.

Table 3.2 The strength of artificial rock concert cubes after 28 days.

Basalt fraction Sizes (mm)	Strength (N/mm^2)
- 4.76 + 2.0	43.6
- 9.52 + 4.76	40.5
- 19.05 + 9.52	39.9

Table 3.3 The strength of matrix materials (Mortar) after 28 days.

No of test	Strength (N/mm^2)
1	22.9
2	20.6
3	18.3

CHAPTER FOUR
TESTING PROCEDURE

4.1 Introduction:

This chapter presents the procedure and the conditions under which this work was carried out.

More than 70 tests were carried out by varying the angle of slope, height of benches and the rock texture to study the effect of rock texture on slope stability.

The aims of these tests were to collect the largest possible number of observation and to observe the behavior of the failure at the open pit mines and quarries.

The results obtained will permit to analyze the effects of the rock texture, the angle of slope and the height of the bench affecting the formulation of the mechanism of failure form of the bench.

4.2 Studied parameters:

The parameters investigated in these tests as provided were as follows:

(I) Grain sizes (-4.76 +2.0), (-9.52 +4.76) and (-19.05 +9.52) mm which resulted in three different textures.

(II) Angle of slope 60°, 75° and 90°.

(III) Two height of bench 7 and 10 cm, (see Table 4.1).

Using these parameters, the following were investigated:

(I) The relationship between the grain sizes and failure loads, for bench height equals 10 cm.

(II) The relationship between the angle of slope and failure loads, for bench height equals 10 cm.

(III) The relationship between the grain sizes and failure loads, for bench height equals 7 cm.
(IV) The relationship between the angle of slope and failure loads, for bench height equals 7 cm.

4.3 Testing equipment:

Uniaxial compression machine was used which is computerized to give slow loading rate to simulate the static load resulted from the rock weights of benches. This machine having three gauges, 20, 60 and 200 tons which show the loads of failure, (see Plate 4.1).

The artificial rock models were tested while they were confined from three sides to be similar to the benches at quarries and open cast mines. They were confined in steel mould, (see Plate 4.2).

4.4 The studied parameter affecting slope stability:

Three parameters were investigated which are of utmost importance. They are the height of the bench which will affect the load of failure of mined rock and the angle of the bench slope. This is controlled usually by the mechanical factors used in the bench design and extraction economy of rock and mineral ores.

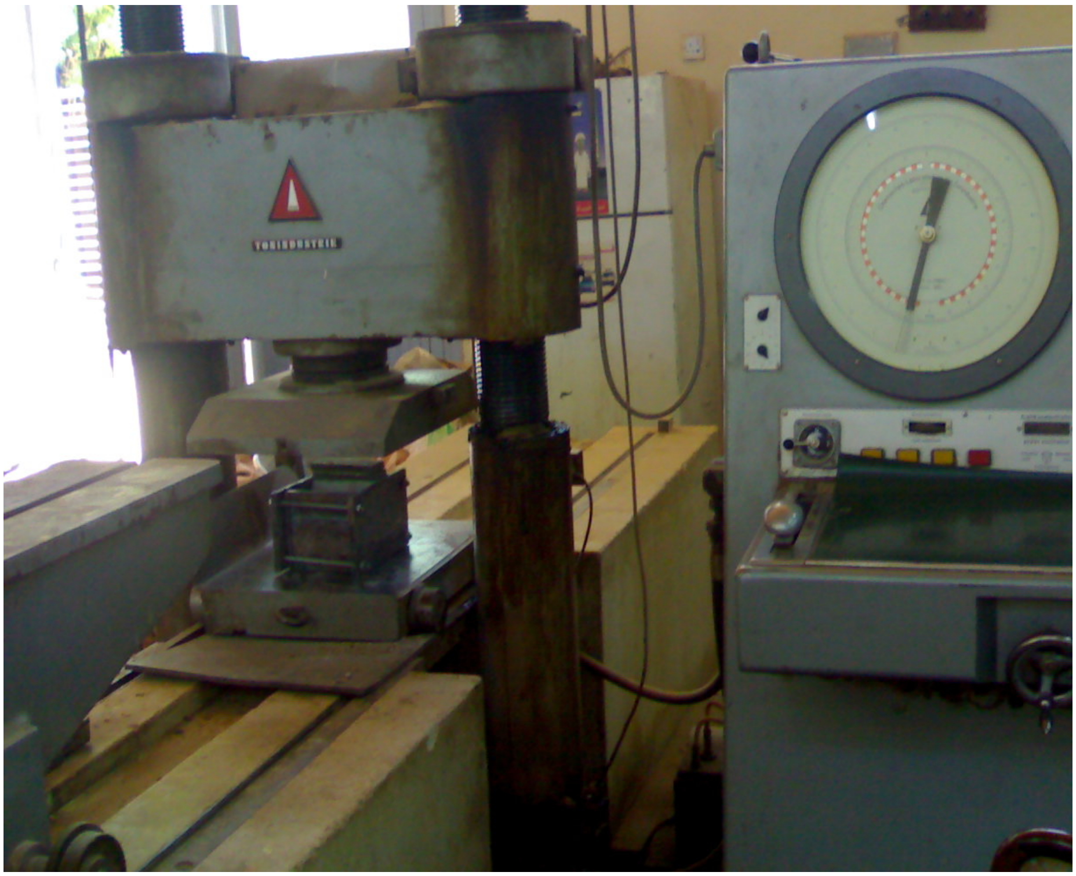


Plate 4.1 Uniaxial compression machine.

Plate 4.2.Shows the specimen models inside the steel moulds during testing.

Table 4.1 shows the studies parameters on the slope stability.

Bench height (cm)	Grain sizes (mm)	Angle of slope (degree)
10	- 4.76 + 2.0	90
10	- 9.52 + 4.76	90
10	- 19.05 + 9.52	90
10	- 4.76 + 2.0	75
10	- 9.52 + 4.76	75
10	- 19.05 + 9.52	75
10	- 4.76 + 2.0	60
10	- 9.52 + 4.76	60
10	- 19.05 + 9.52	60
7	- 4.76 + 2.0	90
7	- 9.52 + 4.76	90
7	- 19.05 + 9.52	90
7	- 4.76 + 2.0	75
7	- 9.52 + 4.76	75
7	- 19.05 + 9.52	75
7	- 4.76 + 2.0	60
7	- 9.52 + 4.76	60
7	- 19.05 + 9.52	60

CHAPTER FIVE

RESULTS AND ANALYSIS

5.1 Introduction

In open pit mines determinations of the slope of benches will solve the most important questions concerning the opening up of the quarry, drilling in mines, mining equipment, etc…

A satisfactory theoretical analysis must account for all the various parameters that control the stability of benches slopes. These parameters could be divided into three groups:

First: Geological parameters, the geology of the materials forming the slope must be sufficiently well delineated so that the lithological boundaries of the different rock types can be determined. This also involves a structural investigation to determine the spatial relationships between the discontinuities;

Secondly: The imposed loading, brought about by benches self-weight (gravity), moisture content, water pressure and seismic loads due to blasting, must be determined;

Thirdly: The available strengths, or alternatively, the allowable displacements or deformations of the different materials forming the slope may be assessed under the loads defined in the second group.

5.2 Parameters Studied in the Present Research Work:

These parameters were the effects of grain sizes, the height of bench and the angle of the slope of the bench.

5.3 Presentation of the Results:

The results of the present work are shown in Table 5.1 to 5.6. These tables show the results of loads causing the failures at various grain sizes of the artificial rock, angle of slope and height of bench.

Table 5.1. Results of failure loads for bench height equals 10 cm and slope angle 60 degree.

Grain Sizes (mm)	Failure Loads (tons)
- 4.76 + 2.0	30.5
- 9.52 + 4.76	25.5
- 19.05 + 9.52	24.9

Table 5.2. Results of failure loads for bench height equals 10 cm and slope angle 75 degree.

Grain Sizes (mm)	Failure Loads (tons)
- 4.76 + 2.0	27.5
- 9.52 + 4.76	23.8
- 19.05 + 9.52	23.4

Table 5.3. Results of failure loads for bench height equals 10 cm and slope angle 90 degree.

Grain Sizes (mm)	Failure Loads (tons)
- 4.76 + 2.0	25.5
- 9.52 + 4.76	22.8
- 19.05 + 9.52	22.4

Table 5.4.Results of failure loads for bench height equals 7 cm and slope angle 60 degree.

Grain Sizes (mm)	Failure Loads (tons)
- 4.76 + 2.0	25.2
- 9.52 + 4.76	24.2
- 19.05 + 9.52	24.0

Table 5.5 .Results of failure loads for bench height equals 7 cm and slope angle 75 degree.

Grain Sizes (mm)	Failure Loads (tons)
- 4.76 + 2.0	22.3
- 9.52 + 4.76	21.5
- 19.05 + 9.52	21.1

Table 5.6. Results of failure loads for bench height equals 7 cm and slope angle 90 degree.

Grain Sizes (mm)	Failure Loads (tons)
- 4.76 + 2.0	22.0
- 9.52 + 4.76	20.8
- 19.05 + 9.52	20.6

5.4 Analysis of the Results:

The obtained results are analyzed and discussed under the following separated headings.

5.4.1 Effect of grain sizes by varying the angles of slope for bench height 10 cm:

The results of the present work are graphically shown in Figures 5.1 to 5.16. Figures 5.1, 5.2 and 5.3 show the effect of grain sizes of the rock on the failure loads for different slope angles and bench height equals 10 cm. It could be read out from the curves on these figures that the rocks which having fine textures (fine grains) will have higher resistance to slide, than those composed of medium and large grains. The increase in the value of angle of slope would decrease the resistance of rock for sliding with all employed grain sizes. The decrease for the fine rock texture is much lower than both medium and large textures. The relationships between the failure loads which represent the weight of the bench resulting in the failure and the angles of slope would give an equations of second order for all types of rock textures.

For angle of slope 60º

$$y = 35.40x^{-0.14}$$

For angle of slope 75º

$$y = 30.95x^{-0.11}$$

For angle of slope 90º

$$y = 27.99x^{-0.09}$$

y = failure loads.

x = grain sizes.

Plate 5.1 shows that no sliding has occurred which indicate that when the angle of slope is equals 60º almost all rock textures resist the sliding. Plate 5.2 gives no slide for fine texture rocks, but the slide occurred with the medium and coarse textures rocks. They may be resulted from both shear and tension stresses,

Plate 5.3 indicates that a circular failure mode due to shearing stresses of materials along failure surface with the different rock textures and angle of slope equals 90 degree.

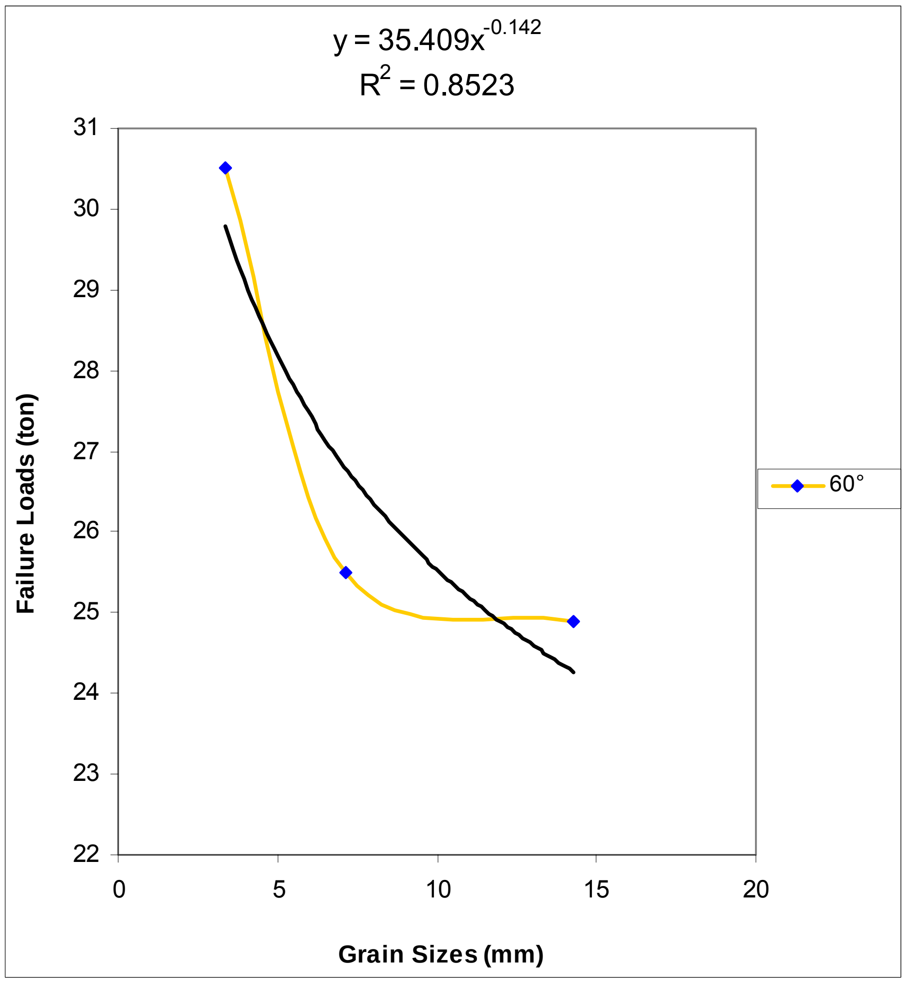


Figure 5.1 shows the relation between failure loads and grain sizes for angle of slope 60° and height of bench 10 cm.

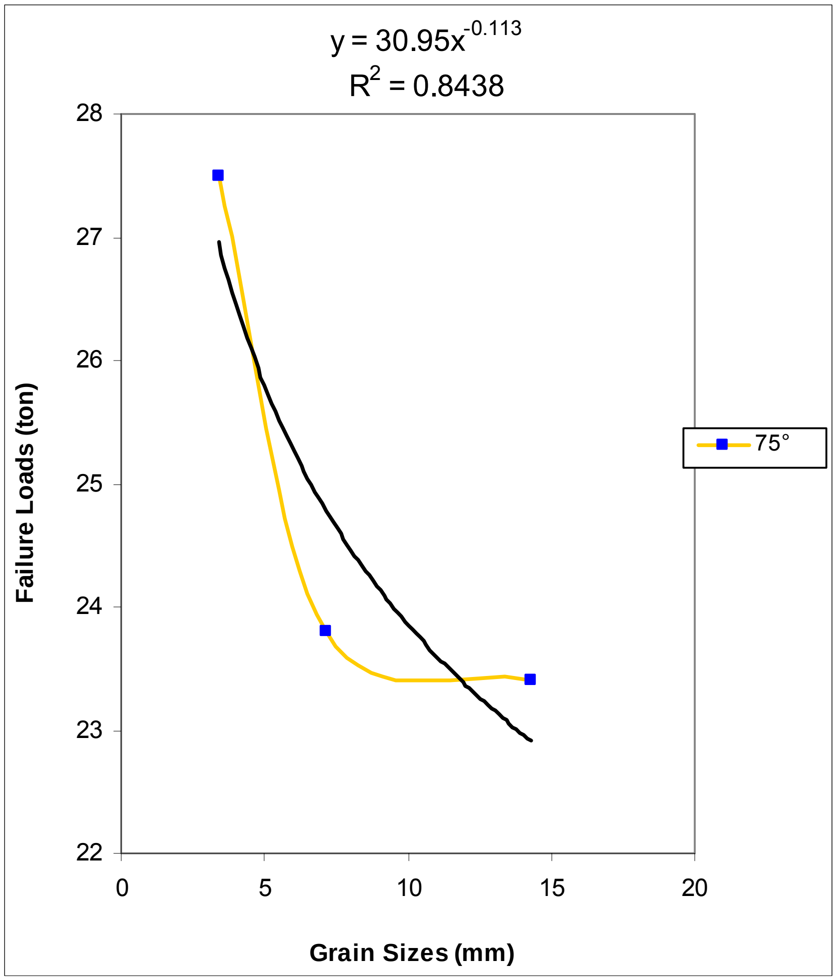


Figure 5.2 shows the relation between failure loads and grain sizes for angle of slope 75° and height of bench 10 cm.

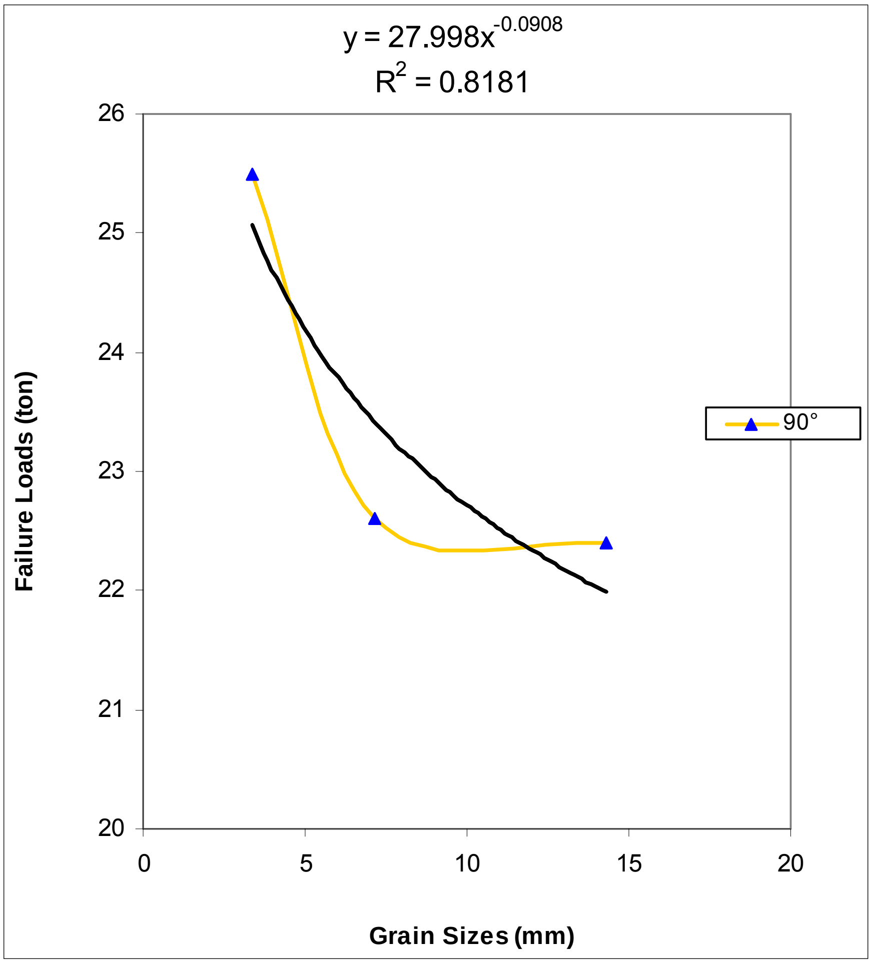


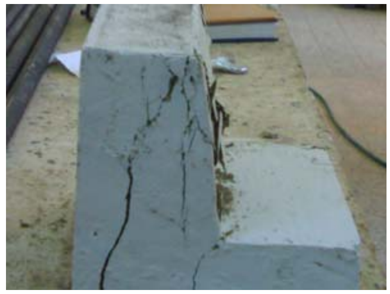
Figure 5.3 shows the relation between failure loads and grain sizes for angle of slope 90° and height of bench 10 cm.

Coarse texture rock

Medium texture rock

Fine texture rock

Plate.5.1. Shows failure modes at angle of slope 60°, for bench height equals 10 cm.



Coarse texture rock

Medium texture rock

Fine texture rock

Plate.5.2. Shows failure modes at angle of slope 75°, for bench height equals 10 cm.

Coarse texture rock

Medium texture rock

Fine texture rock

Plate.5.3. Shows failure modes at angle of slope 90°, for bench height equals 10 cm.

5.4.2 Effect of grain sizes by varying the angles of slope for bench height 7 cm:

Figures 5.4, 5.5 and 5.7 show the relationship between the three types of rocks differing in their grain sizes and failure loads, for angle of slope 60º, 75º and 90º, respectively, however, for bench height equals 7 cm. The results have the same trends of the above summarized results, i.e., the rock resistance to failure by sliding decreases with increasing both the angle of slope and the sizes of rock grains. However, the relationships took second order equations which are shown by the resulted curves.

For angle of slope 60º

$$y = 26.14x^{-0.03}$$

For angle of slope 75º

$$y = 23.31x^{-0.03}$$

For angle of slope 90º

$$y = 23.10x^{-0.04}$$

y = failure loads.

x = fraction sizes.

Plates 5.4 and 5.5 indicate that there is no failure that would be occurred for both these slope angles with the three types of rock textures. Inspection of plate 5.6 however shows that all textures of rock demonstrate circular or near circular failure modes as a result of shear and tension stresses at only the angle of slope 90 degree.

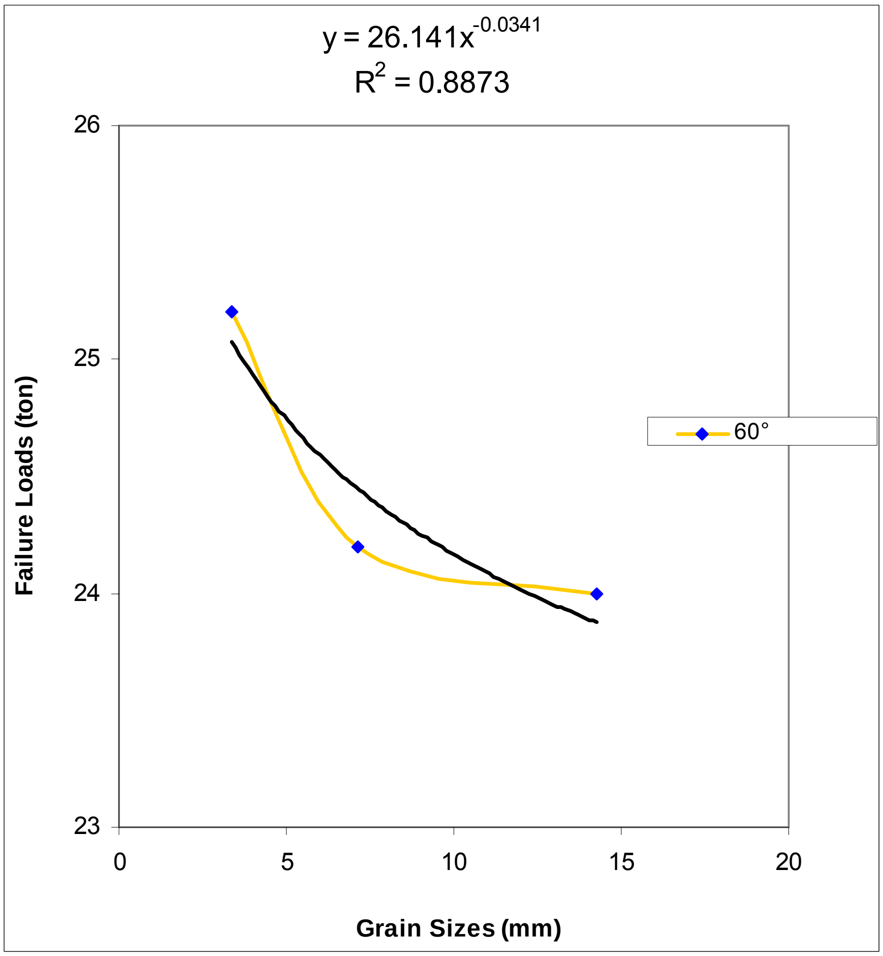


Figure 5.4 shows the relation between failure loads and grain sizes for angle of slope 60° and height of bench 7 cm.

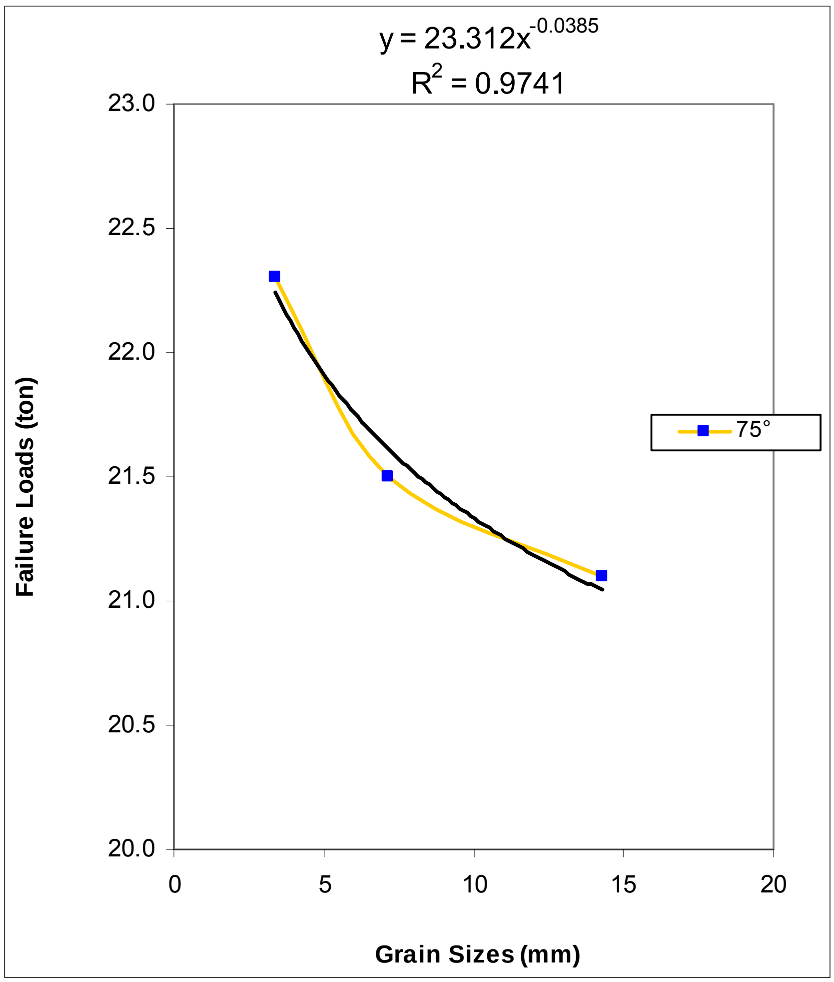


Figure 5.5 shows the relation between failure loads and grain sizes for angle of slope 75° and height of bench 7 cm.

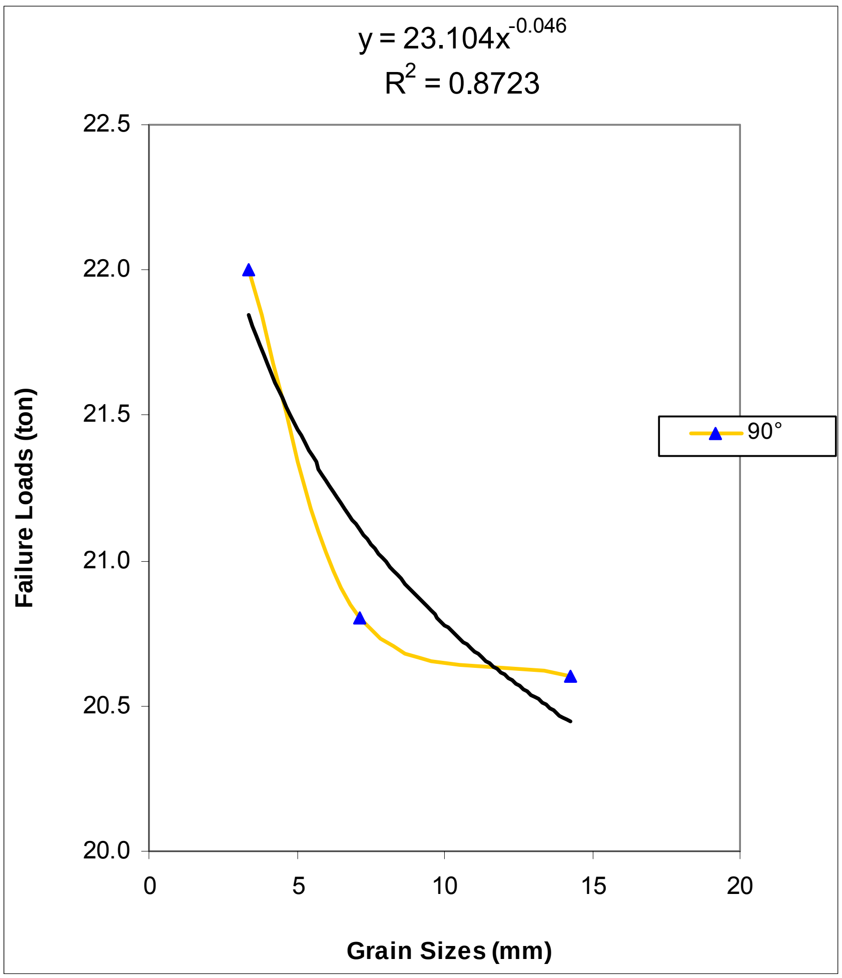


Figure 5.6 shows the relation between failure loads and grain sizes for angle of slope 90° and bench height of bench 7 cm.

Coarse texture rock

Medium texture rock

Fine texture rock

Plate.5.4. Shows failure modes at angle of slope 60°, for bench height equals 7 cm.

Coarse texture rock

Medium texture rock

Fine texture rock

Plate.5.5. Shows failure modes at angle of slope 75°, for bench height equals 7 cm.

Coarse texture rock

Medium texture rock

Fine texture rock

Plate.5.6. Shows failure modes at angle of slope 90°, for bench height equals 7 cm.

5.4.3 Effect of angles of slope by varying the grain sizes for bench height 10 cm:

Figures 5.7 to 5.12 show the relations between the failure loads and both angles of slope and grain sizes of the different rock textures. The relations indicate the same trends as before shown, but all relations are seen as second order relations, (giving curves forms), for the angles of slope and for the various sizes of rock grains.

Figures 5.7, 5.8 and 5.9 give the relationship between different angles of slope and failure loads, for grain sizes of rock (-19.05 + 9.52), (- 9.52 + 4.76) and (- 4.76 + 2.0) mm, respectively, and bench height equals 10 cm.

For grain size (-19.05 mm + 9.52 mm)

$$y = 72.58x^{-0.2616}$$

For grain size (- 9.52 mm + 4.76 mm)

$$y = 86.37x^{-0.29}$$

For grain size (- 4.76 mm + 2.0 mm)

$$y = 186.3x^{-0.44}$$

y = failure loads.

x = angles of slope.

Plate 5.7 and 5.8 present the failure modes for the both cases. No slide resulted in at angle of slope 60º, but circular or nearly circular failure depending on the shear and tension stresses at angle of slope 75º, and also when the angle of slope was increased to 90 degree.

Plate 5.9 shows no failure resulted at angles of slope 60º and 75º, but the circular failure occurred only at 90 degree.

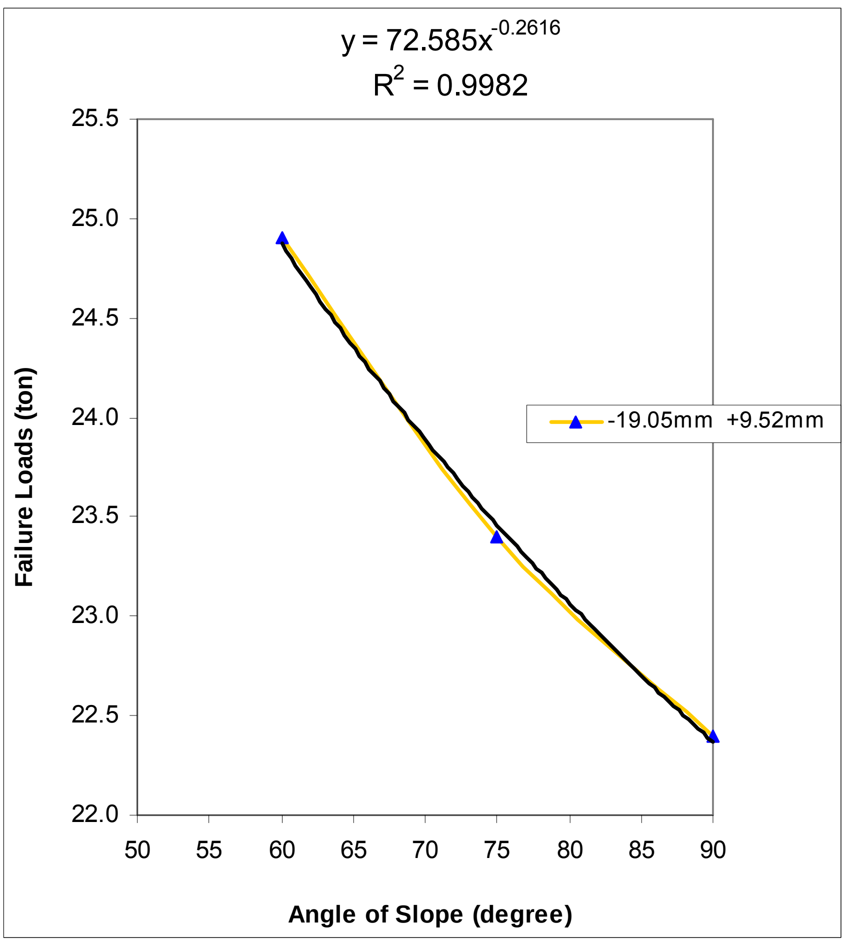


Figure 5.7 shows the relation between failure loads and angle of slope for grain sizes (-19.05 + 9.52 mm) and height of bench 10 cm.

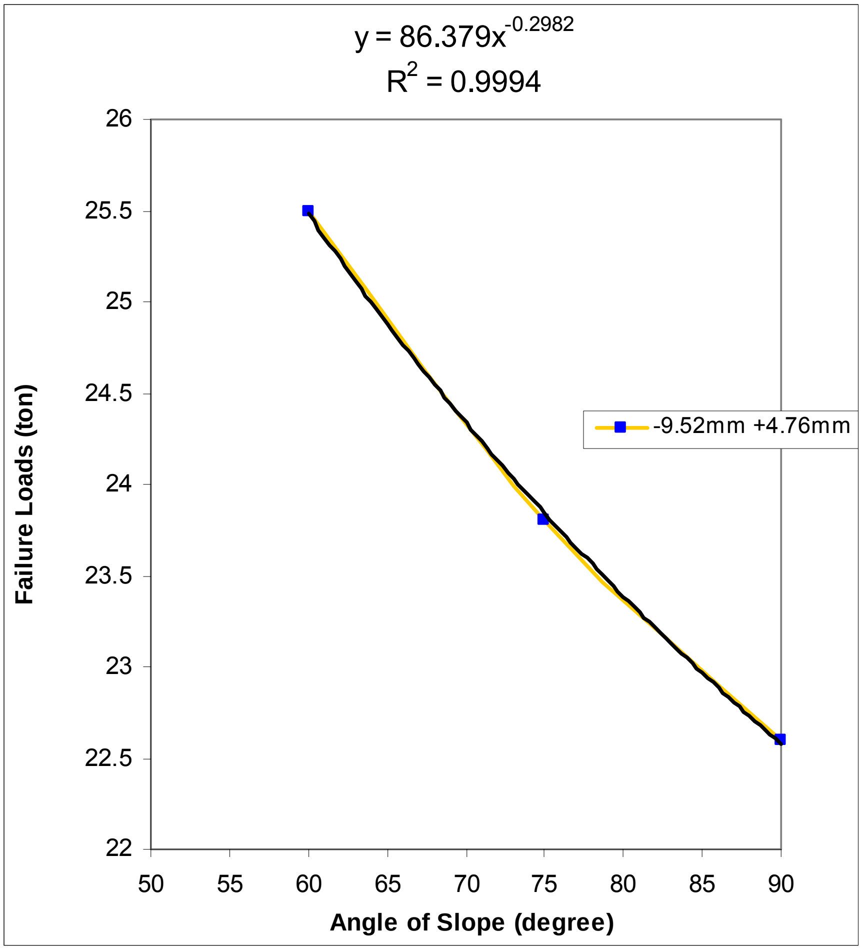


Figure 5.8 shows the relation between failure loads and angle of slope for grain sizes (- 9.52 + 4.76 mm) and height of bench 10 cm.

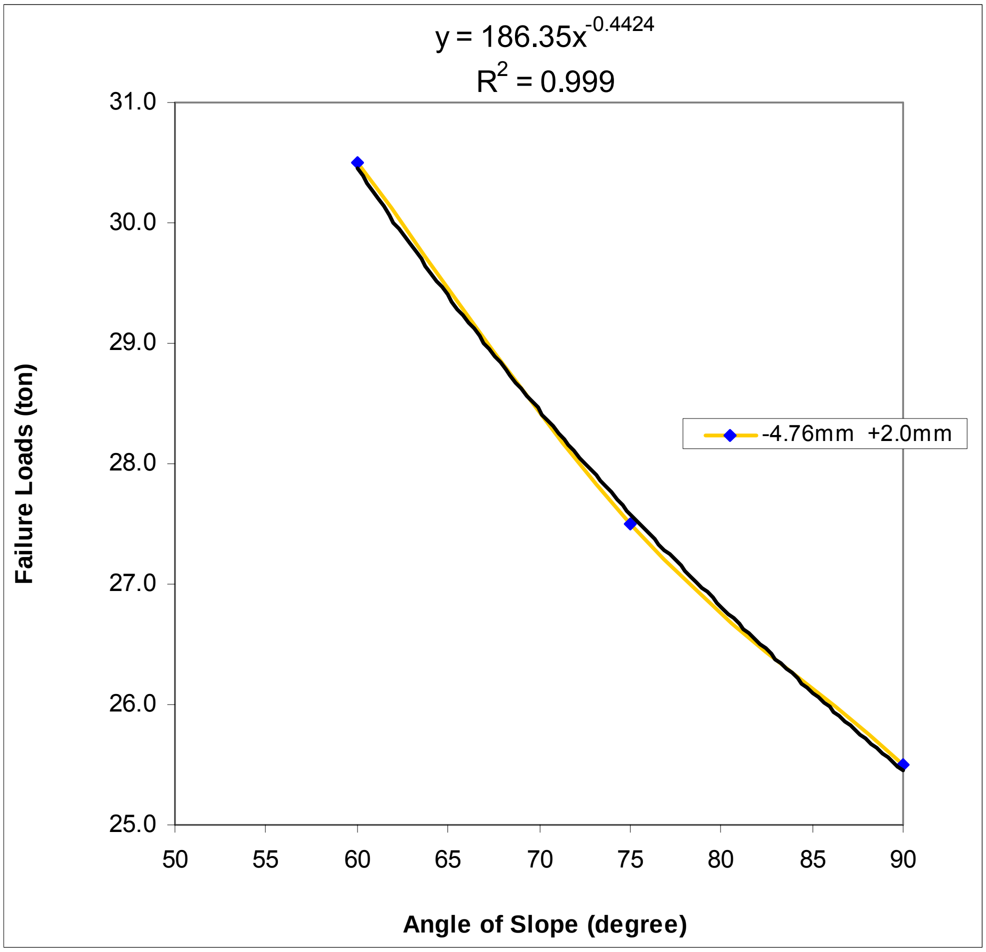


Figure 5.9 shows the relation between failure loads and angle of slope for grain sizes (- 4.76 + 2.0 mm) and height of bench 10 cm.

Angle of slope 60°

Angle of slope 75°

Angle of slope 90°

Plate.5.7. Shows the failure modes at the grain sizes (-19.05 + 9.52 mm) and bench height equals 10 cm.

Angle of slope 60°

Angle of slope 75°

Angle of slope 90°

Plate.5.8. Shows the failure modes at grain sizes (- 9.52 + 4.76 mm) and height equals 10 cm.

Angle of slope 60°

Angle of slope 75°

Angle of slope 90°

Plate.5.9. Shows the failure modes at grain sizes (- 4.76 + 2.0 mm) and bench height equals 10 cm.

5.4.4 Effect of angles of slope by varying the grain sizes for bench height 7 cm:

Figures 5.10, 5.11 and 5.12 show the relationship between the different angles of slope and the failure loads, for grain sizes of rock (-19.05 + 9.52 mm), (- 9.52 + 4.76 mm) and (- 4.76 + 2.0 mm) , respectively, and bench height equals 7 cm.

For grain size (-19.05 mm + 9.52 mm)

$$y = 114.1x^{-0.38}$$

For grain size (- 9.52 mm + 4.76 mm)

$$y = 113.1x^{-0.37}$$

For grain size (- 4.76 mm+ 2.0 mm)

$$y = 101.0x^{-0.3p4}$$

y = failure loads.

x = angles of slope.

Plates 5.10, 5.11 and 5.12 prove that with the slope angle of 60º and 75º no sliding would be occurred, but circular slide could be resulted at angle of slope 90 degree.

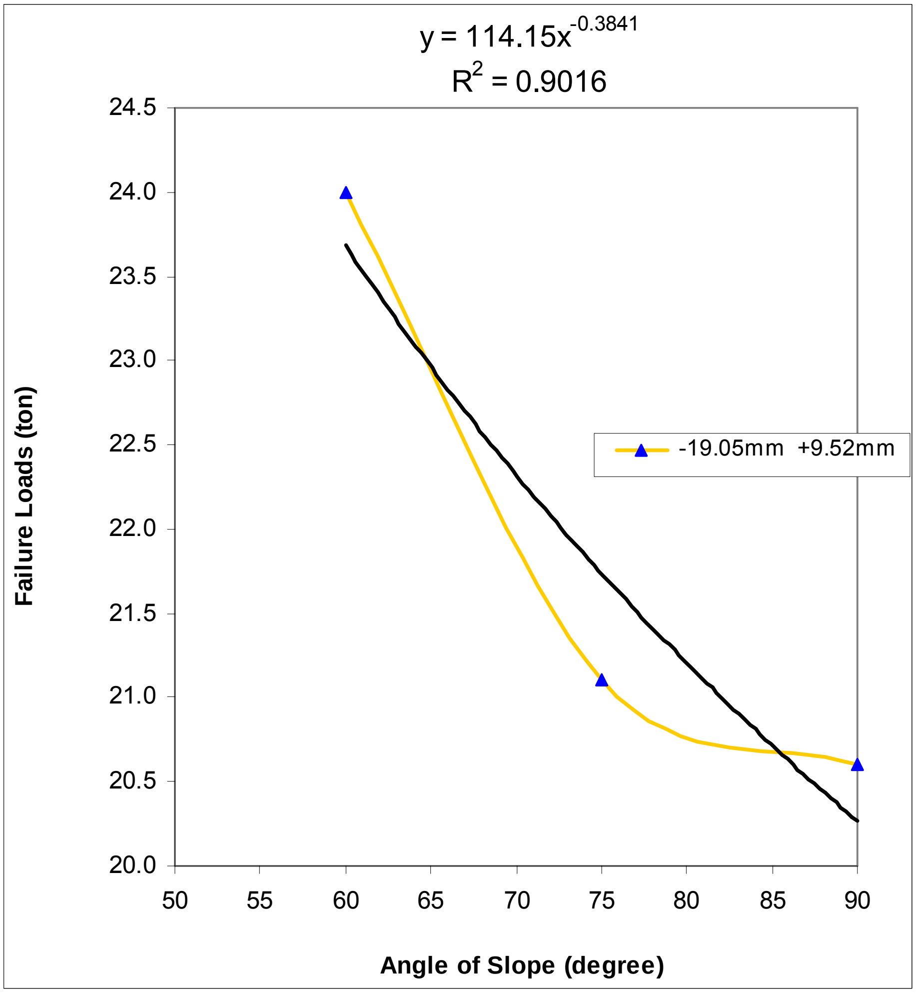


Figure 5.10 shows the relation between failure loads and angle of slope for grain sizes (-19.05 + 9.52 mm) and height of bench 7 cm.

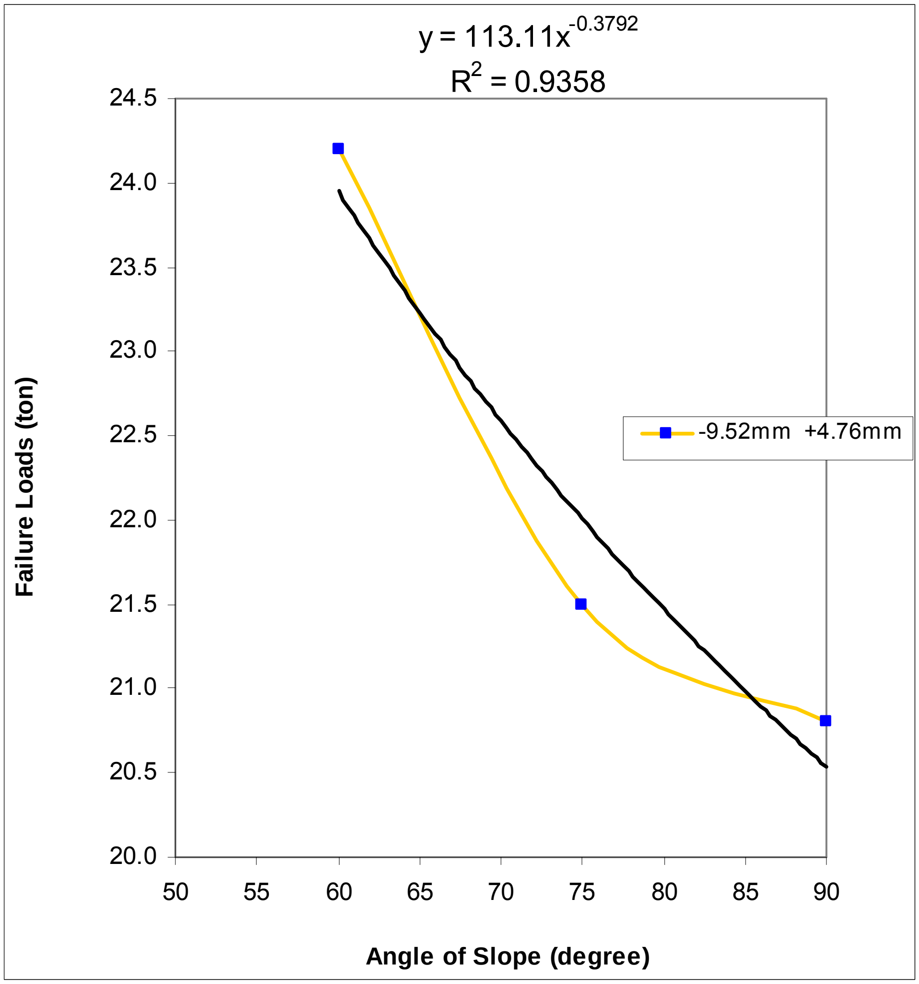


Figure 5.11 shows the relation between failure loads and angle of slope for grain sizes (- 9.52 + 4.76 mm) and height of bench 7 cm.

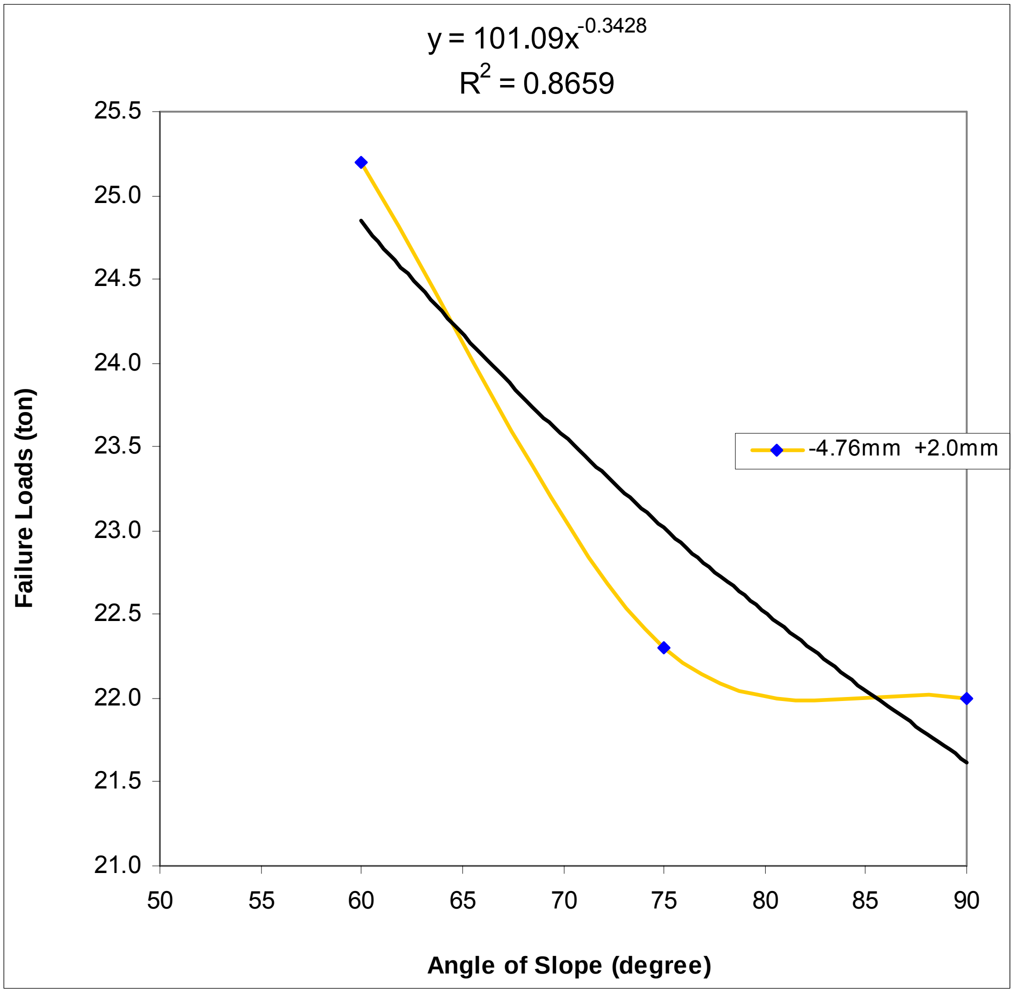


Figure 5.12 shows the relation between failure loads and angle of slope for grain sizes (- 4.76 + 2.0 mm) and height of bench 7 cm.

Angle of slope 60°

Angle of slope 75°

Angle of slope 90°

Plate.5.10. Shows the failure modes for rock having grain sizes (-19.05 + 9.52 mm) and bench height equals 7 cm.

Angle of slope 60°

Angle of slope 75°

Angle of slope 90°

Plate.5.11. Shows the failure modes for rock having grain sizes
(- 9.52 + 4.76 mm) and bench height equals 7 cm.

Angle of slope 60°

Angle of slope 75°

Angle of slope 90°

Plate.5.12. Shows the failure modes for rock having grain sizes (- 4.76 mm + 2.0 mm) and bench height equals 7 cm.

An explanation which could be given here for the above described results, with the fine texture rocks, where the gains will have larger surface area than both medium and coarse textures, therefore, the cohesion between the grains and the matrix materials will be greater for fine grains than that between the matrix materials and medium and coarse grains, Hence the resistance to failure is greater for fine grains than that for both medium and coarse ones.

In addition, the shapes of coarse and medium grains are shown to take flake shapes, while the shapes of fine grains are nearly cubic. Referring to rock mechanics principles, rocks having coarse and flake shape grains will have a low resistance to breaking than that composed of fine grains. These could give clearly explanation for the analyzed results.

In figures 13 to 16 show a comparison between figures 1 to 3, 4 to 6, 7 to 9 and 10 to12, respectively.

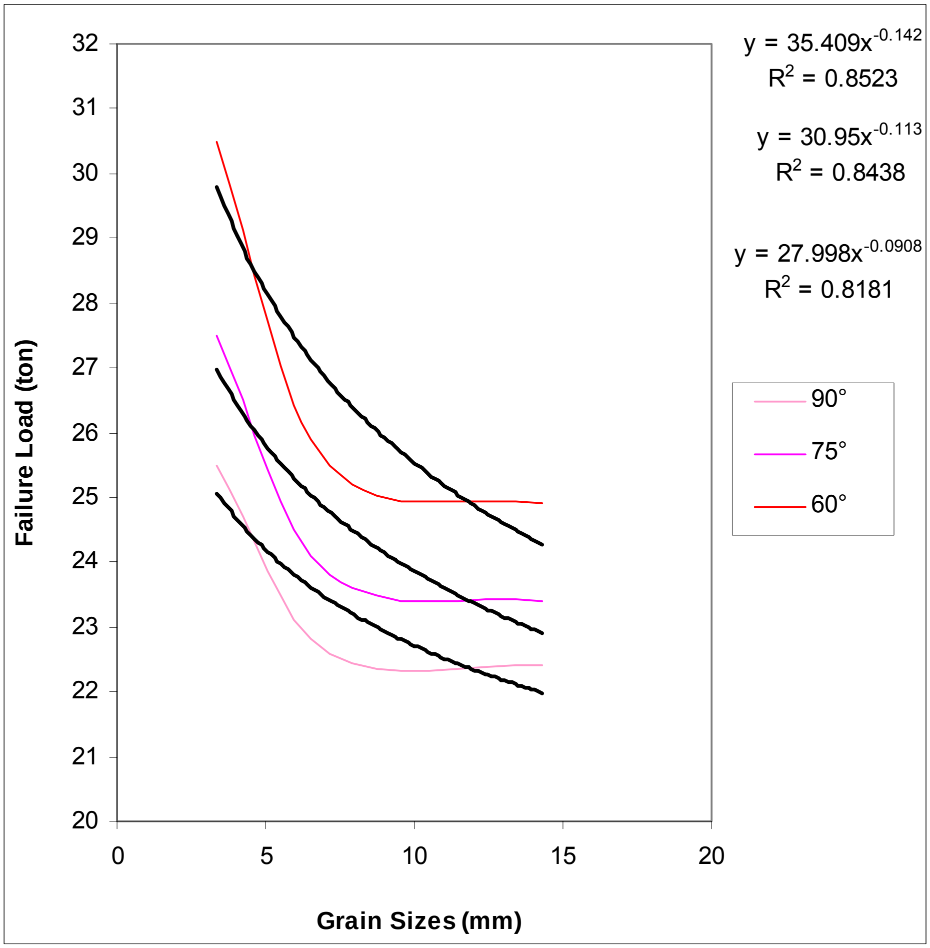


Figure 5.13 shows the relation between failure loads and different grain sizes for different angle of slope and height bench 10 cm.

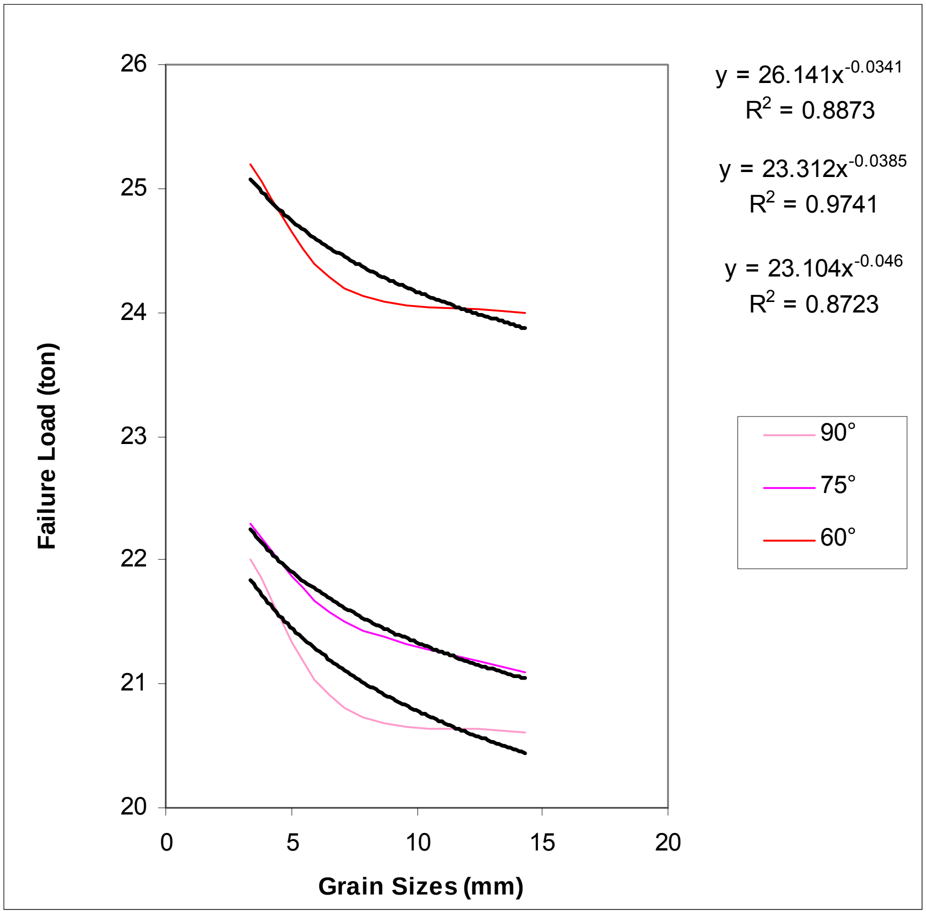


Figure 5.14 shows the relation between failure loads and different grain sizes for different angle of slope and height bench 7 cm.

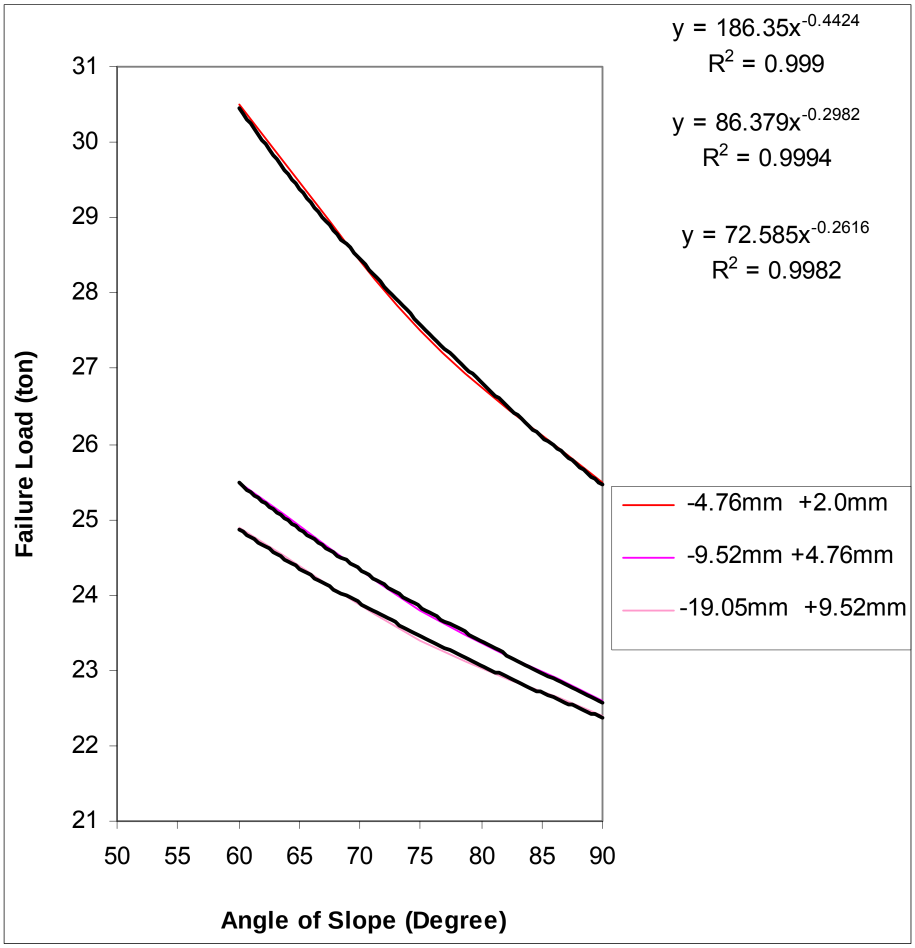


Figure 5.15 shows the relation between failure loads and different angle of slope for different grain sizes and height bench 10 cm.

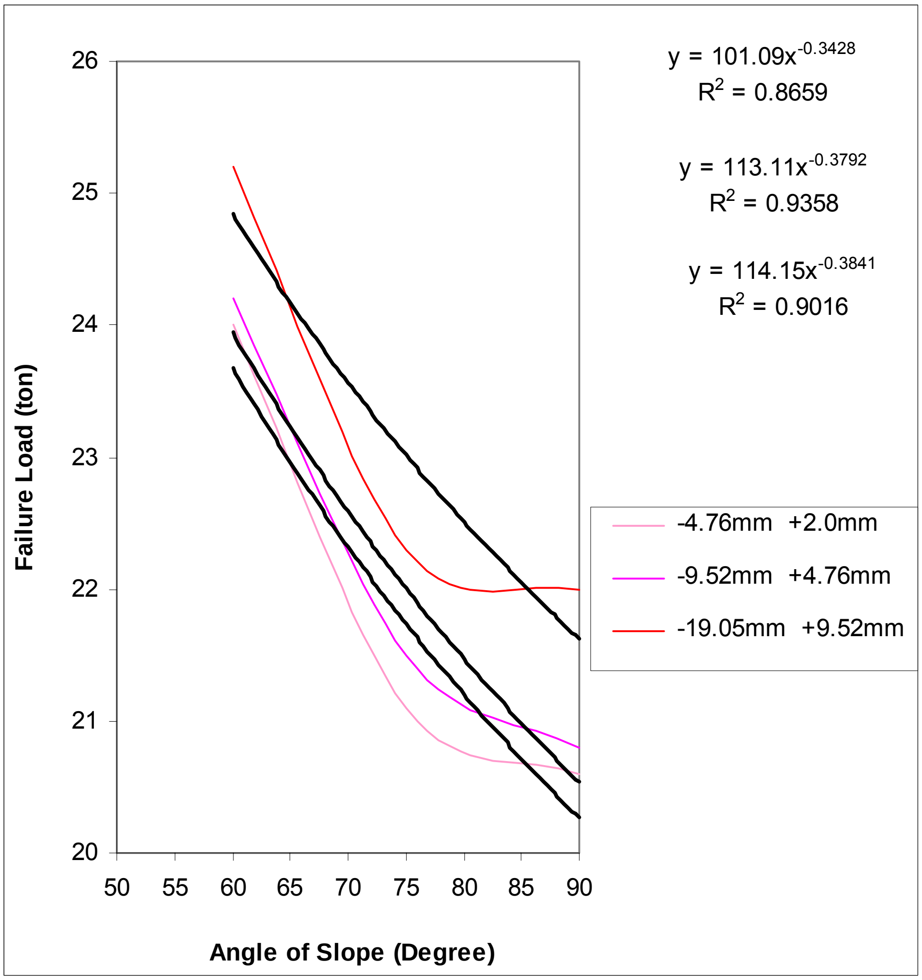


Figure 5.16 shows the relation between failure loads and different angle of slope for different grain sizes and height bench 7 cm.

CHAPTER SIX

Conclusions and Recommendations

6.1 Conclusions:

This study deals with the stability of bench slope in mining industry (quarries and open pits). Physical models were made and tested under confined compressive strength to simulate the practices (the benches are made in the deposits being extracted).

A review for the various published works on this topic has shown that many parameters were studied and reported in these published articles. However, it was appeared that some of parameters have not been dealt with such as the influence of rocks textures combined with the bench height and slope. Therefore, these parameters were studied together in the present thesis.

Three types of artificial rocks varying in their textures were made using grains from basalt which has larger compressive strength than the matrix material bound these grains together to yield the artificial models.

The rock textures resulted are characterized as coarse (- 19.05 + 9.52 mm), medium (- 9.52 + 4.76 mm) and fine textures (- 4.76 + 2.0 mm). Also the angles of slope were studied 60, 75 and 90 degree.

The following conclusions could be withdrawn from the obtained results:

- The fine texture rocks gave higher resistance to the failure, even at angles greater than 75 degree, but before the slope angle reaches 90 degree.

- For the medium texture rocks, the benches would fail at angles of slope less than 75° even at decreasing the height of benches. This is due to tension and shear stresses which accrued with textures having medium grain and matrix having lesser compressive strengths.
- The coarse texture rocks gave a resistance to the failures only at angle of slope 60 degree, i. e., at angle of slopes are greater than 60 degree the benches will fail.

6.2 Recommendations:

The following point should be considered for the future researches in this area:

- The artificial rocks should be made from sand having different sizes, i. e., coarse, medium, fine and very fine sands, since the sands affect the matrix of the artificial rocks.
- The grains composing the rocks should be selected on their strengths, i. e., using natural rock having softer, medium and strong strengths. For example using sandstone, limestone, marble, feldspar, etc…
- Study the effects moisture content with the different artificial rocks.

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